ASEISMIC CAPABILITY OF A LARGE TURBINE BUILDING UNDER SEVERE EARTHQUAKES

K. Muto(I), T.Tsugawa(I), Y. Takenaka(I)
S.Sato(II), K.Toyama(III), S.Kamagata(IV)
Presenting Author: Y. Takenaka

SUMMARY

The aseismic capability of an actual turbine building is discussed herein through experimental and analytical investigations. As the building is composed of braced frames and non-braced frames, a method of nonlinear dynamic analysis, in consideration of buckling of bracings, is developed after the structural tests of braced frame. Floor distortions between adjacent frames are also considered in the analysis. Referring to the experimental findings, maximum story drift and maximum deformation of bracings obtained by the computer analysis are considered to confirm that the building is able to maintain its safeness during and after a severe earthquake.

I. INTRODUCTION

This paper describes investigations on aseismic capability of an actual turbine building for a 1000 MWe thermal power plant. The building consists of a number of 5-storied steel frames as shown in Fig.1. Bracings in the form of a letter K are placed between the wide flange columns and the beams as the main aseismic element. These bracings have been suspected of losing the load bearing capacity once a large story drift occured during earthquakes. Another particular problem regarding the structural feature is that the building contains large void spaces in the vertical cross section as well as large openings in the floors, which means that the floor cannot be treated as a rigid element. It is, therefore, necessary to take into account the different movements of the constitutive frames especially in the transverse direction.

As the qualifying work of the turbine building (Ref.1), structural tests of the K-shaped braced frame is conducted, at first, using actual design features. On the basis of the test results and its simulating method, a computer program is developed for the dynamic analysis of the whole structure, where plane frames are connected to each other by springs representing floor components. After introduction of improved numerical computation, the nonlinear responses under a severe earthquake with maximum intensity of 400 gal are computed and the aseismic capability of the turbine building is evaluated.

(I) Muto Institute of Structural Mechanics, Inc.
(II) Tokyo Electric Power Service Co., Ltd.
(III) Kajima Institute of Construction Technology, Kajima Corporation
(IV) Information Processing Center, Kajima Corporation

II. LOADING TEST OF BRACED FRAME

Outline of Testing

The K-shaped frame of 1/2 scaled size is fabricated and tested by cyclic horizontal loading as shown in Fig.2. This test specimen accurately corresponds to actual design regarding all the members as well as the details of connecting portion with gusset plates and high tension bolts as shown in Fig.3. Horizontal forces are alternately applied at the top of columns using oil jacks until the story drift angle reaches 1/30 at the final stage. Load-displacement curve obtained is shown in Fig.4. Although initial buckling of compressive bracing is observed in the second cycle, it is found that the loss of load bearing capacity of the frame is small and that the stable spindle-like hysteresis behavior is maintained until large deformations.

Simulation of Testing

By coupling the generalized plastic hinge models for columns and beams with an empirical hysteresis model for bracings proposed by Wakabayashi (Ref.2), the authors analysed the test results and strived to simulate the loading procedures. The simulated load-displacement curve is finally obtained as shown in Fig.5. It is also found that the nonlinear behavior of the frame after buckling is fairly identified by the analysis. The small difference at the initial stage between test and analysis is attributed to the model of bracing which is rather oriented towards post-buckling behavior.

III. NONLINEAR DYNAMIC ANALYSIS OF BRACED FRAME STRUCTURE

Analytical Model

Adopting the same hysteresis models as in the test simulation, a computer program is developed for nonlinear dynamic analysis of the building during earthquakes. In order to take into account of the effect of interaction between adjacent frames, the constitutive plane frames of the building are connected to each other by springs representing floor components. Different movement of the building portion is, then, represented by this modeling, where, the mass of the building and equipments is discretized and lumped at each joint of column, beam and/or bracing.

Equation of Motion

Equation of motion at time t in terms of incremental horizontal and vertical displacements (ΔU, ΔV) under earthquake excitation $\{\Delta\ddot{\alpha}\}$ is introduced using mass matrix [M] for each plane frame as follows;

$$\begin{bmatrix} M & 0 \\ 0 & M \end{bmatrix} \begin{Bmatrix} \Delta\ddot{U} \\ \Delta\ddot{V} \end{Bmatrix} + \begin{bmatrix} Kuu & Kuv \\ Kvu & Kvv \end{bmatrix} \begin{Bmatrix} \Delta U \\ \Delta V \end{Bmatrix} + \begin{Bmatrix} \Delta Bu \\ \Delta Bv \end{Bmatrix} + \begin{Bmatrix} \Delta F \\ 0 \end{Bmatrix} = - \begin{bmatrix} M & 0 \\ 0 & M \end{bmatrix} \begin{Bmatrix} \Delta\ddot{\alpha} \\ 0 \end{Bmatrix} \quad \ldots\ldots \ (1)$$

where [K] denotes the tangential stiffness matrix of the plane frame except all bracing members while {B} and {F} are the force vectors due to bracings and floor springs respectively.

Introducing the trapezoidal rule and eliminating the vertical displacements of Eq.(1) yields the following reduced equation, in terms of horizontal displacements as;

$$[M]\{\Delta\ddot{U}\} + [Ku]\{\Delta U\} + \{\Delta Bu\} + \{\Delta F\} - [H1]\{\Delta Bu\}$$
$$= - [M]\{\Delta\ddot{\alpha}\} - [H2]\{L\} \quad \dots\dots\dots\dots (2)$$

where

$$[Ku] = [Kuu] - [Kuv][G]^{-1}[Kvu], \quad [H1] = [Kuv][G]^{-1}$$
$$[H2] = [Kuv] - [Kuv][G]^{-1}[Kvv], \quad [G] = \frac{4}{\Delta t^2}[M] + [Kvv]$$
$$\{L\} = \Delta t\{\dot{V}o\} + \frac{\Delta t^2}{2}\{\ddot{V}o\} .$$

Numerical Computation

Unknown vectors {U}, {B} and {F} at a certain time t in Eq.(2) are obtained by the following iterations: that is, in order to solve the Eq.(2) at the S-th step of iteration process, the equation

$$[M]\{\Delta\ddot{U}\}^S + [Ku]\{\Delta U\}^S = -[M]\{\Delta\ddot{\alpha}\} - [H2]\{L\}$$
$$-\{\Delta Bu\}^{S-1} - \{\Delta F\}^{S-1} + [H1]\{\Delta Bv\}^{S-1} \quad \dots\dots (3)$$

is calculated using the knowns obtained at the (S-1)th step until the error E_R defined by

$$E_R = \sum_i \mid \ddot{U}_i^S - \ddot{U}_i^{S-1} \mid / \sum_i \mid \ddot{U}_i^S \mid \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (4)$$

attains the prescribed allowable limit. After the convergence of the above computation, stresses of all members are computed and, if necessary, rearrangement of stiffness matrix [K] is performed.

IV. ASEISMIC CAPABILITY OF TURBINE BUILDING

Results of Analysis

Free vibration modes are computed and illustrated in Fig.6. The first, second and third natural period is 0.666, 0.310 and 0.230 sec. respectively. The first and second modes represent the assembly of the respective mode of individual frames, while deformed shapes similar to torsion of the whole structure are observed in the third mode.

Dynamic response of the building is computed using earthquake wave of the El Centro 1940 NS with the maximum intensity of 400 gal provided that the damping factor is 2% for the first natural period. The maximum acceleration and shearing forces which occurred in the individual frames are shown in Fig.7, which are enveloped expression obtained in the form of time history and of restoring-force as shown in Fig.8.

Evaluation of Aseismic Capability

The qualitative value of story drift and inelastic deformation of structural members are important indices to measure the nonlinear state of the structure. According to Table 1, the largest value of maximum story drift angle R is 1/113 in the 4th story of 338 frame. The value suggests that, referring to the test results, each frame is expected to maintain both strength and deformability. Defining the ductility factor of the bracing by the ratio of maximum axial deformation to tensile yielding elongation, the largest ductility factor is 4.2 occured in the right side bracing of the 5th story of 320 frame as listed in the table. From the comparison of the ductility factor $6.0 \sim 7.0$ observed at R=1/100 in the testing, it is expected that all bracings are still sound enough to resist earthquake forces without particular deterioration of load bearing capacity. It is, therefore, confirmed that the building will sustain severe earthquakes having maximum intensity of 400 gal.

Design Consideration

From a point of view of structural design, it is noteworthy that the responses of constitutive frames result in appreciable differences to each other. This reason is attributed to the fact that the nonlinear deformations are concentrated due to deficiency of floor stiffness and to unbalanced arrangement of bracings. Since such structural layout is, however, more or less unavoidable because of the mechanical demands for turbines and generators, the design of braced frames must be executed with attention given to the transmission of earthquake forces throughout the whole structure.

V. CONCLUDING REMARKS

As the qualifying work of an actual large turbine building, loading test and nonlinear dynamic analysis are performed. The dynamic analysis method developed in these investigations is capable of tracing the nonlinear state of the structure due to buckling and yielding. It is confirmed that the turbine building possesses sufficient capability against a severe earthquake. In case of braced frame structure where nonlinearity of the frame and the relative deformation between adjacent frames are not negligible, the rigorous dynamic design using effective dynamic analysis method is most desirable. The authors believe that the outcomes of this paper can lead to the establishment of a more reliable design procedure.

REFERENCES

(1) Kiyoshi Muto et al., "Experimental and Analytical Study of Braced Frame Structure", Proc. of 28th Symposium on Structural Engineering, Tokyo, 1982.

(2) Minoru Wakabayashi et al., "Hysteretic Behavior of Steel Braces Subjected to Horizontal Load due to Earthquake", Proc. 6th WCEE, New Delhi, India, 1977.

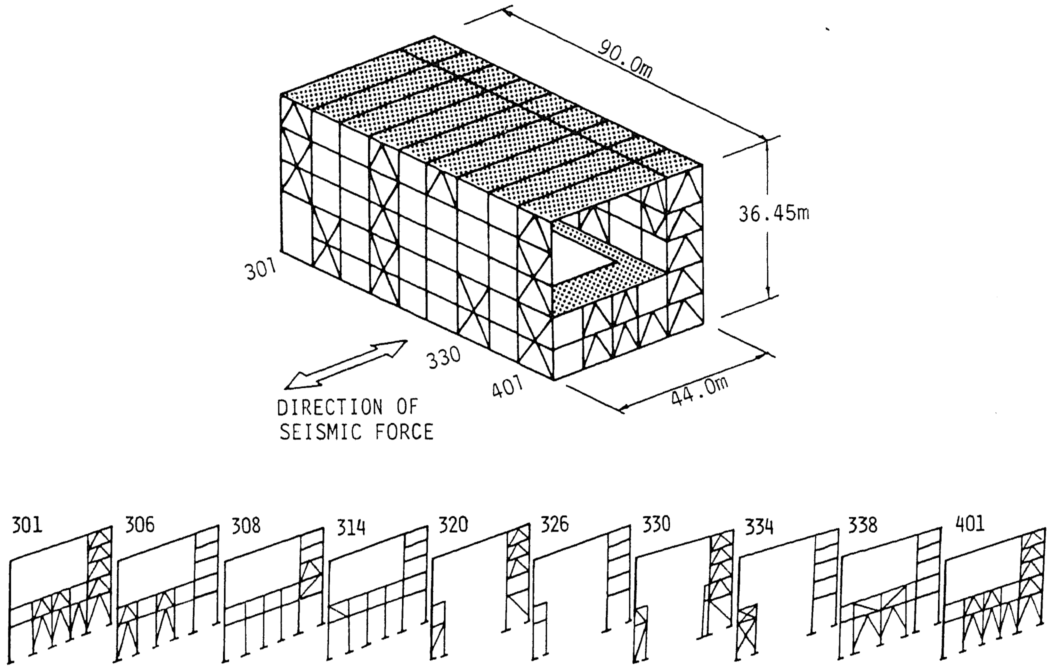


FIG.1 SCHEMATIC VIEW OF TURBINE BUILDING AND CONSTITUTIVE FRAMES IN TRANSVERSE DIRECTION

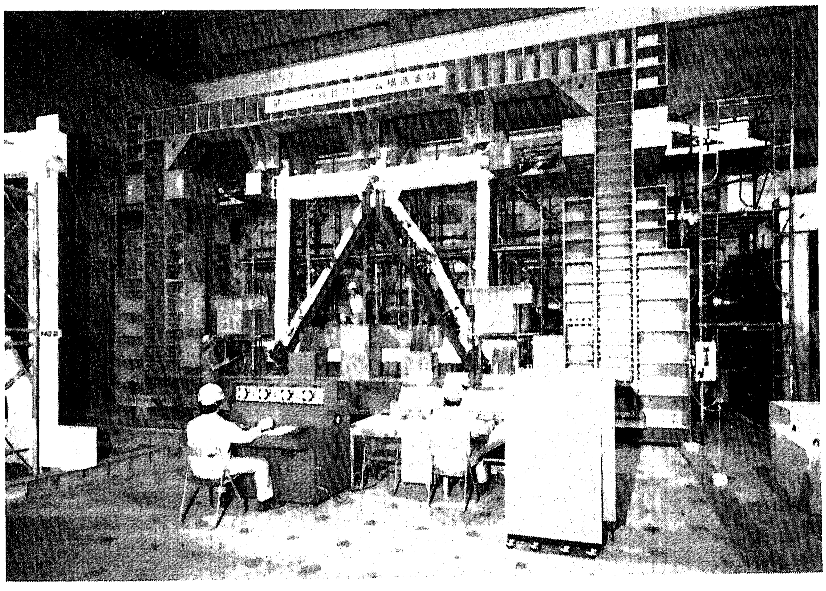
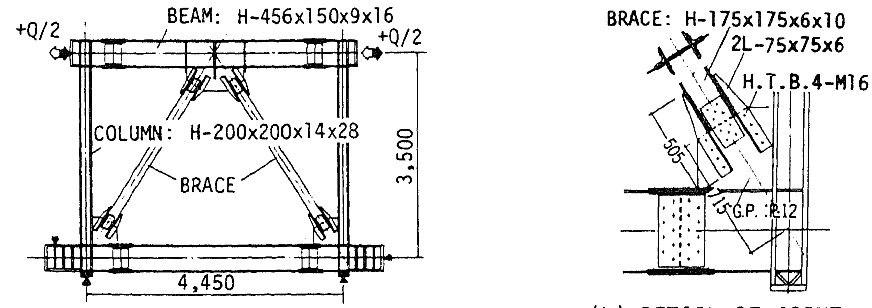


FIG.2 LOADING TEST OF K-SHAPED BRACED FRAME



(a) BRACED FRAME OF 1/2 SCALED SIZE (b) DETAIL OF JOINT

FIG.3 TEST SPECIMEN

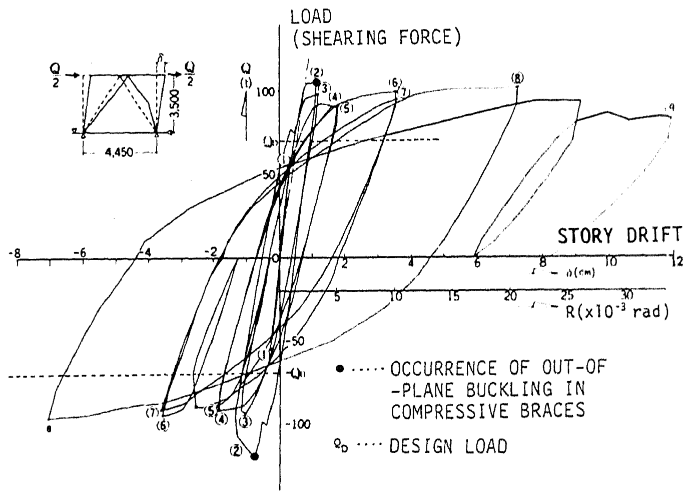


FIG.4 Q-δ CURVE OF SPECIMEN OBTAINED BY TESTING

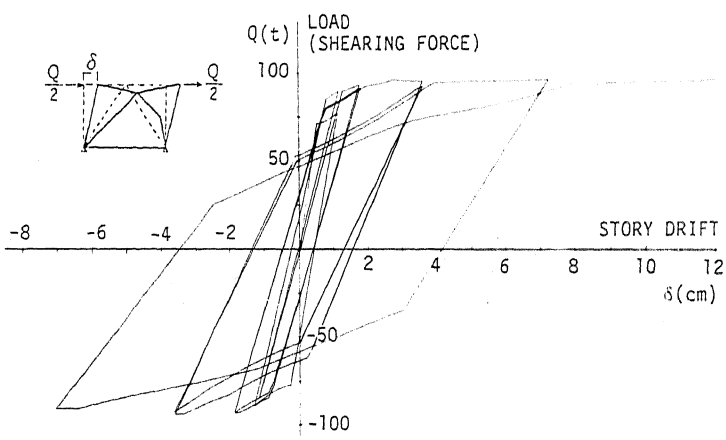
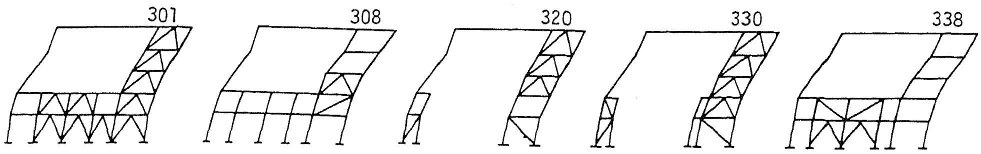
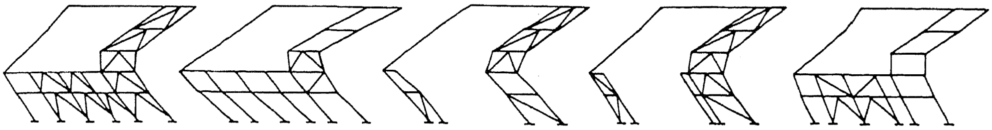


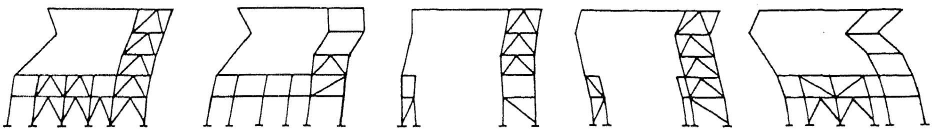
FIG.5 Q-δ CURVE OBTAINED BY SIMULATION ANALYSIS



(a) 1st MODE (T_1=0.666sec β_1=1.600)

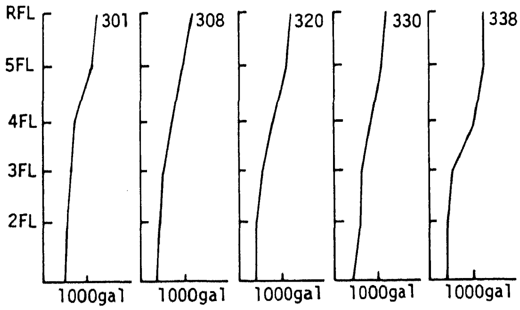


(b) 2nd MODE (T_2=0.310sec β_2=-0.899)

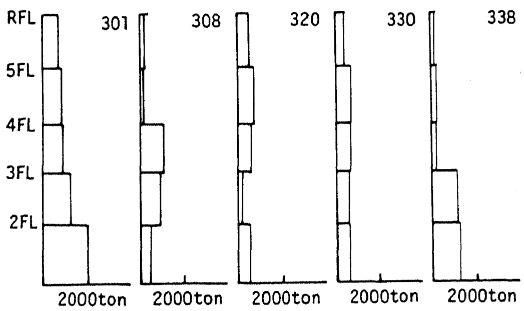


(c) 3rd MODE (T_3=0.230sec β_3=-0.155)

FIG.6 FREE VIBRATION MODE

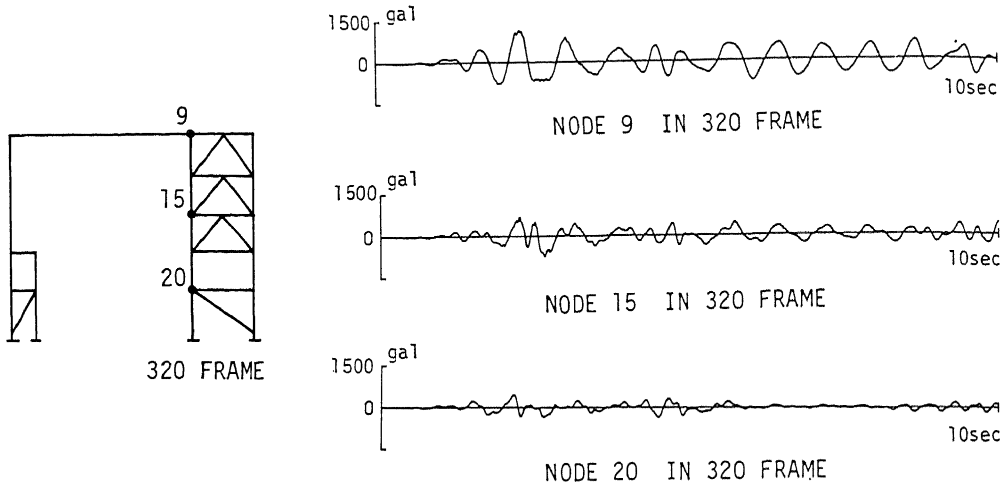


(a) MAXIMUM ACCELERATION

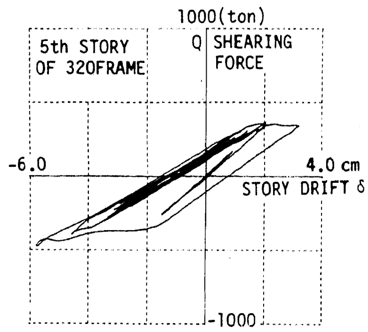


(b) MAXIMUM SHEAR

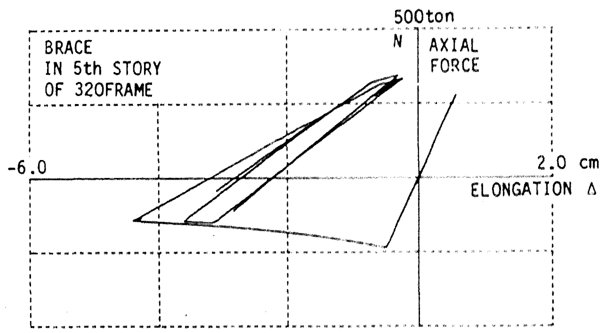
FIG.7 DISTRIBUTION OF MAXIMUM RESPONSES
(EL CENTRO 1940 NS A_{MAX} = 400 gal)



(a) TIME HISTORY OF HORIZONTAL ACCELERATION



(b) BEHAVIOR OF 5th STORY IN 320 FRAME



(c) BEHAVIOR OF BRACING OF 5th STORY IN 320 FRAME

FIG.8 EXAMPLES OF RESPONSE TIME HISTORY (EL CENTRO 1940 NS A MAX = 400 gal)

TABLE.1 RESPONSE DUE TO THE EL CENTRO EARTHQUAKE (A MAX = 400 gal)

STORY	MAXIMUM STORY DRIFT ANGLE (rad.)	MAXIMUM DUCTILITY FACTOR OF BRACE
5th	1/178(338FRAME)～1/124(301FRAME)	0.51(401FRAME)～4.16(320FRAME)
4th	1/163(301FRAME)～1/113(338FRAME)	0.60(301FRAME)～2.83(401FRAME)
3rd	1/278(308FRAME)～1/159(338FRAME)	0.62(301FRAME)～2.76(330FRAME)
2nd	1/420(338FRAME)～1/238(320FRAME)	0.41(338FRAME)～2.19(330FRAME)
1st	1/547(401FRAME)～1/341(308FRAME)	0.48(330FRAME)～1.56(301FRAME)